STRUCTURAL RESPONSE MONITORING OF NEW ZEALAND BRIDGES

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ABSTRACT

In 2010, New Zealand introduced High Productivity Motor Vehicles onto the road network. To date, the movement of these heavier vehicles has been limited by the capacity of infrastructure, particularly bridges on the State Highway and local road networks. To limit the extent of costly bridge strengthening and replacement, the New Zealand Transport Agency is currently undertaking Structural Response Monitoring of three key bridges in the South Island. The Structural Response Monitoring systems employed to date include a range of conventional bridge monitoring and testing techniques, such as visual inspections, material testing, and survey levelling; as well as more advanced monitoring systems using accelerometers, displacement transducers, vehicle weigh-in-motion testing, advanced bridge model calibration, and concrete condition assessment. Whilst the monitoring is still its initial stages, significant conclusions around the structural performance of the bridges have already been made. These include the low likelihood of first degree resonance of the beams, calculation of a bridge specific impact factor, information on the continuity of the joints and interactions of the split piers. To minimise the cost of response monitoring, a five step methodology was developed. This included preliminary diagnostics, detailed bridge analysis and model calibration, assessment of critical failure mechanisms and focussed response monitoring on key regions under high stress. This paper outlines the findings of this testing and provides a cost effective solution for the monitoring of bridge structures.

KEYWORDS

Structural response monitoring, High Productivity Motor Vehicle, bridge diagnostics, simply supported, advanced bridge model calibration, critical failure mechanisms, risk mitigation.

BACKGROUND

In May 2010, the New Zealand Ministry of Transport amended the Vehicle Dimension and Mass (VDM) Rule. This amendment allowed vehicle operators with divisible loads to apply for 'High Productivity Motor Vehicle' (HPMV) permits to operate on approved routes at greater dimension and mass limits than those that would otherwise be allowed under the Rule. However, HPMV movements have been restricted by the capacity of key infrastructure, particularly weaker bridges on the State Highway (SH) and Territorial Local Authority (TLA) road networks.

There are approximately 15,500 road bridges in New Zealand, with 4,500 bridges and large culverts on the New Zealand Transport Agency (NZ Transport Agency) State Highway Network and around 11,000 on TLA roads. Whilst most bridges constructed since 1943 have the capacity to support HPMV loading; design standards prior to the 1940's had considerably lower live load demands, meaning that many bridges constructed prior to this period cannot support HPMVs without some form of strengthening. A significant percentage (around 18%) of New Zealand's road bridges were constructed prior to 1943, as is the case for much of the developed world. Therefore, there is a significant cost in upgrading infrastructure to support these heavier vehicles.

Between 2010 and 2012, significant investigations were undertaken to identify the restrictive bridges on key freight networks, re-analyse and test bridges to provide more accurate structural data, quantify the freight demand on these networks, and determine which routes could economically be strengthened. In 2013, the NZ Transport Agency embarked on the first significant phase (Tranche 1) of strengthening of bridges on key investment routes to allow them to support HPMV loading. The key South Island route over SH1 between Christchurch and Oamaru was identified as a Tranche 1 investment route due to the considerable amount of freight transported over this highway. Three significant impediments to the upgrade of this route were the Rakaia and two Rangitata River Bridges.

INTRODUCTION

The Rakaia River and Rangitata River No.1 and No.2 Bridges are located on State Highway 1S (SH1S) within the South Island of New Zealand, just north of Rakaia township, and approximately 35km south of Ashburton township respectively. All three bridges were constructed between 1939 and 1940, and comprise of conventional two lane reinforced concrete bridges, with essentially identical superstructures. The superstructures consist of four reinforced concrete T-beams, supported on reinforced concrete columns and pile caps, founded on driven reinforced concrete piles, as illustrated in Figure 1 below.

At 1.76km long, the Rakaia River Bridge is the longest bridge in New Zealand, and consists of 144, 40 foot (12.2m) spans. The Rangitata River No.1 and No.2 Bridges are 650m and 320m long and consist of 53 and 26, 12.2m spans respectively. All three bridges have expansion joints in the form of split piers generally every 5 spans. The bridges are currently managed by Opus International Consultants Ltd (Opus) as part of the Region 11 and 12 (Canterbury and West Coast) Bridge Management Contract. Previous assessments of the bridges undertaken by Opus indicated that they are all under-capacity for supporting Full HPMV loading.

Figure 1 View of SH1S Rakaia River Bridge beams, superstructure and piers

Given the length of the Rakaia and Rangitata River Bridges; the cost of strengthening these bridges to support HPMV loading was estimated at over $10M (NZD). Therefore, the NZ Transport Agency and Opus have proposed allowing these bridges to operate at slightly higher stresses, as allowed for within Section 7.4.3 of NZ Transport Agency Bridge Manual (the Bridge Manual). However, this constitutes as a departure from the Bridge Manual, as clause 7.4.3 (i) requires the bridges to be one of a small number of bridges restricting vehicles on an important route, and clause 7.4.3 (vi) requires early replacement or strengthening to be feasible. Given the number of bridges, the high cost and time required for strengthening and the lack of feasible alternative routes, these criteria are not considered to be met.

As such, Opus have developed a Structural Response Monitoring (SRM) system. This includes a range of conventional bridge monitoring and testing techniques, such as visual inspections, material testing, crack monitoring and survey levelling; as well as more advanced monitoring systems using accelerometers, displacement transducers, vehicle weigh-in-motion testing, advanced bridge model calibration, and concrete condition assessment. The objectives of SRM are:

1. To provide confidence in the HPMV load capacity and safety of the bridges; and
2. To provide confidence that the bridges will not require unexpected, early replacement or extensive refurbishment due to serviceability problems.

This paper provides a cost effective solution to monitoring bridge structures and outlines the findings from the monitoring undertaken to date.

STRUCTURAL RESPONSE MONITORING (SRM) METHODOLOGY

Historically, Structural Health Monitoring (SHM) of bridges has typically been limited to larger more complex structures. Detailed monitoring of more simple, low value structures is often cost-prohibitive, due to the cost of electronic equipment, access for installation, maintenance and ongoing storage, processing and interpretation of data.

To mitigate risk and minimise costs, a five step SRM methodology has been developed for the monitoring of the Rakaia and Rangitata River Bridges, as shown in Figure 2 below. This methodology has the potential to provide significant whole life cost improvements to older bridge structures by avoiding, reducing or delaying costly strengthening or bridge replacement. This is particularly the case where observed deterioration is less than would be expected from analysis, or where significant unknowns exist in the assessment and modelling of the structure. The monitoring also provides valuable data on the strength and performance of the most common type of weaker bridge in New Zealand, thereby assisting the management of other typical bridges.

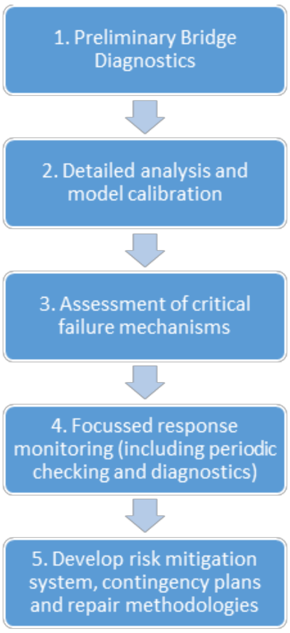

Figure 2 Proposed Structural Response Monitoring (SRM) Methodology

STRUCTURAL RESPONSE MONITORING (SRM) – CASE STUDY

Case Study of the Rakaia and Rangitata River Bridges

The five step SRM methodology outlined in Figure 2 has been employed on the Rakaia and Rangitata River Bridges between June 2014 and July 2015. Details of the monitoring systems utilised are described in the sections below. Although the monitoring system has only been in place for a short period of time, the preliminary findings have already provided some significant conclusions for these bridges.

Preliminary Bridge Diagnostics

The purpose of preliminary bridge diagnostics is to better understand the condition and behaviour of a bridge to help refine the assessment models. Various techniques were utilised on the Rakaia and Rangitata River Bridges, as described below. Many of these also provided long term response monitoring systems on the bridge.

Concrete condition assessment

An extensive concrete condition assessment of was undertaken on the Rakaia River Bridge in October 2014. This included a comprehensive visual inspection, sounding of the concrete to identify areas of spalling, a reinforcement cover meter survey, x-ray fluorescence spectroscopy (XRF) testing to measure chloride ion ingress, pH testing to assess the depletion of $Ca(OH)_2$ and estimate the carbonation depth, and isolated concrete breakout to identify the extent of corrosion and condition of reinforcing bars. The extent of carbonation and chloride ion ingress was compared with the depth of reinforcement measured, to assess the overall risk of corrosion to the bars. This information was then be used to assess the likely maintenance demand on the bridge, and expected remaining life.

The concrete investigations confirmed that the bridge was generally in good condition for its 76 year age. Concrete testing results showed that the environment was very benign, with very low corrosion rates observed. The carbonation front was not generally at the level of the outer reinforcing bars, and chloride ingress was found to be minimal, with only very low risk of initiating corrosion at average cover depths. Based on the test results, minor increases in crack widths from heavier loading are unlikely to significantly reduce the life of the bridge.

The worst concrete deterioration was observed in the upper sections of the split pier columns, beneath the partially split deck joints, and in the soffit of the deck cantilevers. This damage was attributed to poor compaction of the concrete within these zones, combined with significant bending moments in the split piers. Similar conclusions would be expected for the Rangitata River Bridges, which from visual inspections appear in comparable (slightly better) condition to the Rakaia River Bridge.

Material testing

Opus Research undertook non-destructive steel leeb hardness testing on 35 samples taken from all three bridges to derive the yield strength of the reinforcing steel. Given the close proximity of the bridges, the similar construction dates and similarities in the sample sets, these results can be considered as a larger combined sample set. The average yield stress of the reinforcement was calculated as 288MPa, compared with a previously assumed value of 205MPa from the NZ Transport Agency Bridge Manual. Statistical assessment indicated that the lower 5th percentile yield strength for the whole sample set was 254MPa for a single bar, 265MPa for a group of two bars, and 279MPa for a group of 12 reinforcing bars at the mid-span of the beams.

Concrete core samples were also taken and crushed to determine the material properties of the concrete. Concrete compressive strength results for the Rakaia River Bridge were highly variable, ranging from 17.0 – 62.0MPa with an average of 35.7MPa. This was likely due to the variability the quality of materials and the quality of construction and compaction of the concrete. Samples with rounded greywacke aggregate were identified in three of the sample spans, and exhibited lower strengths. Samples with crushed greywacke aggregate were identified in the other two sample spans, and exhibited considerably higher strengths.

Survey levelling

Conventional survey was undertaken on the bridges using total stations to calculate the level of the mid-span of each of the beams. This provided an indication of typical beam sagging along the length of the bridge, and acted as a benchmark for future monitoring. The survey indicated that the largest variation in levels occurs in spans adjacent to split piers, indicating that some of the split piers may have undergone minor plastic rotation. More sophisticated three-dimensional survey from beneath the bridges is proposed in the future to reduce the need for lane closures during survey, and provide a more holistic survey profile of the bridges.

Accelerometer instrumentation

Three 'Reftek' uni-axial accelerometers and one tri-axial accelerometer were installed at the mid-span of all four beams on the 6th span from the northern abutment of the Rakaia River Bridge in October 2014. All accelerometer outputs are sampled at 200Hz, and the data acquisition system connected to mains power adjacent to the bridge. These acceleration outputs were then double integrated to calculate real-time deflection of the beams at mid-span. This was undertaken to allow the bridge model to be more accurately calibrated, and to provide ongoing monitoring of the response of the beams to increased live loading. A further benefit of having a tri-axial accelerometer installed is the ability to analyse the bridge's performance during seismic events, which still occur on a regular basis within the Canterbury Region.

Accelerometers also have the benefit of providing the fundamental frequency of the bridge span based on the residual vibrations of the beams following impact loading. This can be used to back-calculate a number of structural parameters, including beam stiffness and end fixity conditions. The fundamental frequency of vibration on the Rakaia River Bridge was found to be in the order of 9Hz, compared with a theoretical fundamental frequency of 3-4Hz as a simply supported span or 7-9Hz with fixed supports. This confirmed that the spans have considerable continuity/fixity at their ends. Based on the measured fundamental frequency and typical truck axle spacings, heavy vehicle speeds in excess of 130km/hr would be required to initiate resonance in the spans, and this is considered highly unlikely on these narrow bridges. This therefore reduces the likelihood of impact factors reaching the level of 1.3 specified in the Bridge Manual.

Test loading

A number of cycles of test loading were undertaken on all three bridges using truck combinations that loaded the bridge spans to their maximum legal (Class 1) limits. This testing measured mid-span beam deflection and allowed the accelerometers to be calibrated to enable beam deflection to be derived from acceleration.

The results from one of the more critical cycles of test loading are summarised in Table 1 below. The maximum beam displacements under two lanes of Class 1 loading at 10km/hr were around 3.7mm. This compares with theoretical simply supported displacements in excess of 10mm, indicating that considerable continuity existed over the pier supports. The difference in displacement between adjacent beams allow the beam and deck stiffness values to be more accurately calculated.

A span specific impact factor was calculated based on the sum of the beam displacements compared with the base displacement at crawl speed (<10km/hr). The sum of the displacements has been used, as individual beam displacements are highly dependent on transverse vehicle position. The results show that the Rakaia River Bridge span has a maximum impact factor of around 1.18 under this particular vehicle load scenario. Obviously, considerably more vehicle runs would be required to give this result more statistical accuracy. However, initial results indicate that the Bridge Manual impact factor of 1.3 may be conservative for the Rakaia River Bridge. Moving forward, a system is being developed to calculate impact by correlating results from an adjacent Weigh-in-Motion (WiM) site with the response of the bridge span.

Table 1: Mid-span beam displacement from the live load testing of the Rakaia River Bridge

Vehicle Run	Displacement (dial gauge)					
	Beam 1	Beam 2	Beam 3	Beam 4	Total	Impact Factor
	mm	mm	mm	mm		
10kph nose-tail	2.551524	2.399312	1.378563	0.387255	6.716654	1.000
30kph nose-tail	2.376332	2.478641	1.484802	0.442470	6.782245	1.010
50kph nose-tai	2.358729	2.443023	1.484382	0.456998	6.743132	1.004
70kph nose-tail	2.431792	2.683347	1.676344	0.522231	7.313714	1.089
90kph nose-tail	2.525382	2.939504	1.901973	0.587374	7.954233	1.184
10kph side/side	2.915932	3.704854	3.672480	2.274259	12.56753	1.000
30kph,side/side	3.002314	3.640685	3.620911	2.164330	12.42824	0.989
90kph side/side	3.401528	4.142769	4.133257	2.447413	14.12497	1.123

Figure 3 below shows the typical mid-span displacement response of each of the Rakaia River Bridge beams under the test load travelling north. This further proves the continuity between spans, with loads in adjacent spans resulting in uplift at mid-span. The test loads also show interaction between full depth split piers. The horizontal load and bending in one split pier causes bending in the adjacent pier, resulting in minor uplift in the adjacent span (approximately 0.25mm uplift shown after the downwards deflection in figure ~~10~~ 3 below). This interaction has now been incorporated in our calibrated bridge model, and has an effect on the overall capacity and performance of the bridge.

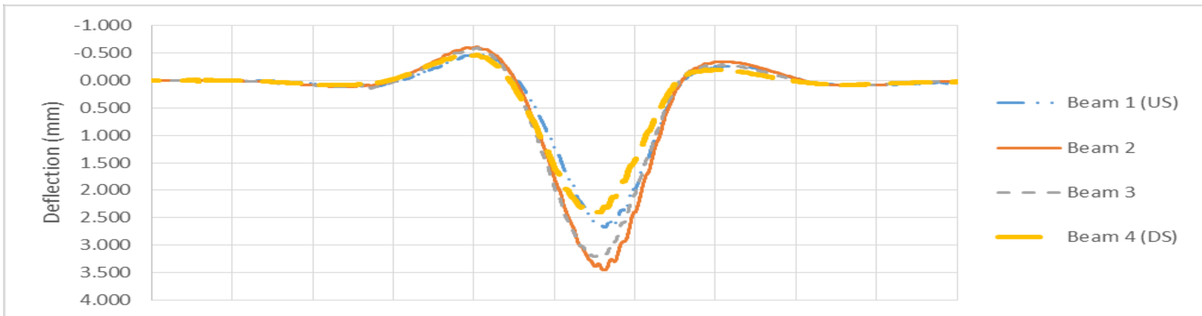

Figure 3: Typical vertical displacement vs time response at mid-span under side-by-side test load runs. This is for span 6 from the northern end, which has a split pier at one end.

Detailed Analysis and Model Calibration

One significant benefit of the SRM and test loading of the bridge was the ability to refine the analysis model of the bridge. The following calibration was undertaken:

- Material testing was used to calculate more accurate properties for the concrete and reinforcing steel.
- The true level of fixity at the ends of the beams under serviceability loads was modelled, based on the fundamental frequency of the bridge span and the measured mid-span displacements under test loads.
- The variation in beam stiffness could be calculated, based on the distribution of mid-span deflection under test loading of one and both lanes. This allowed the contribution of the kerbs and handrails to beam stiffness to be determined.
- The true stiffness of the deck under serviceability loads was calculated, based on the deflection of the beams under the unloaded lanes during test loading.
- The level of continuity through the partially split piers was calculated, based on the known vertical uplift in beams during the test loading.
- The level of continuity between split pier columns was calculated, based on the known vertical uplift in beams during the test loading.
- The range of expected thermal movement at the joints was calculated by applying a thermal profile to the microstran grillage model.

This information was used to calibrate and refine a three-dimensional grillage model of the bridge, as shown in Figure 4, to allow it to accurately predict the true deflection and stresses in critical elements.

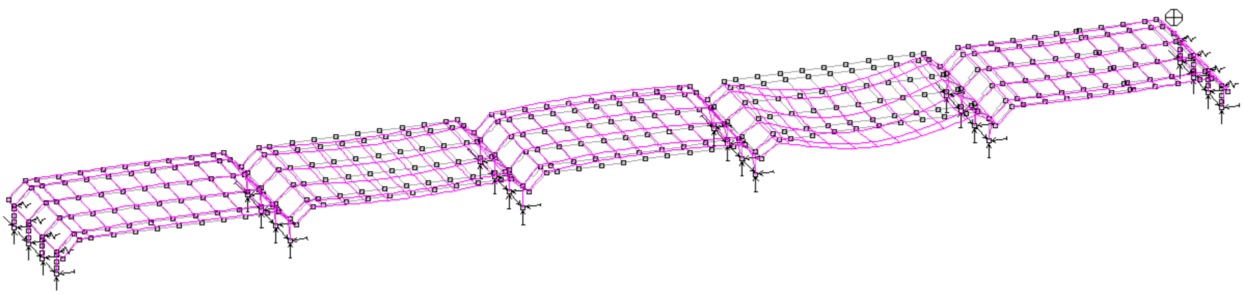

Figure 4 Three-Dimensional Grillage Model of Rakaia and Rangitata River Bridges. Typical deflection profile from vehicle movement is shown.

Assessment of Critical Failure Mechanisms

Previous analysis of the Rakaia and Rangitata River Bridges indicated that they had live load capacities of only 87% HPMV, limited by the mid-span bending capacity of the beams. All previous bridge assessments assumed that the beams act as simply supported at the Ultimate Limit State. This assumption was based on the detailing of the steel in the ends of the beams, and curtailment of all longitudinal beam reinforcement, as shown in Figure 5. However, with a small amount of fixity at the pier supports, the mid-span bending moments decrease significantly, resulting in increases in the overall capacity of the bridge. Conversely, fixity though the pier joints has the potential to cause significant stresses within reinforcing steel in these zones. This increases the risk of damage to the joints, and yielding and/or fatigue failure in reinforcing bars that were not designed to resist these high loads. Given these joints were found to exhibit considerable fixity; further detailed assessment of the loads within the joints was undertaken.

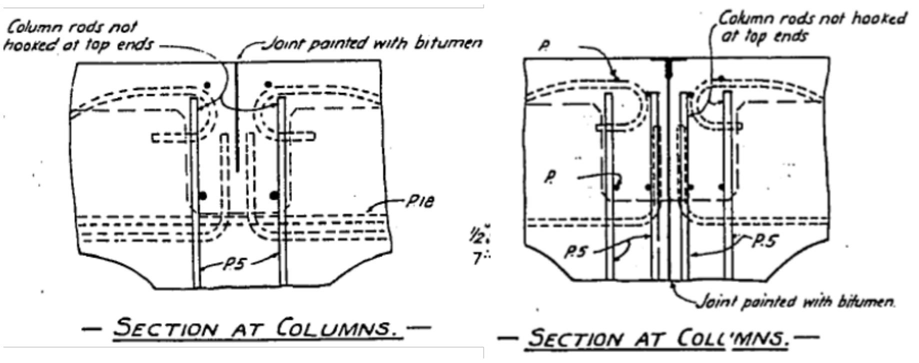

Figure 5: Typical beam reinforcement at pier and split pier joints of Rakaia and Rangitata River Bridges.

Outputs from the grillage model were inputted into Computer-Aided Strut-and-Tie (CAST) software, to assess critical components of the beam-column joint, as shown in Figure 6. Given the complexity of the reinforced concrete joint; a simple finite element model was developed within the software programme LUSAS to confirm that the strut-and-tie geometry accurately represented the elastic concrete stress geometry in the joint, as shown in Figure 7.

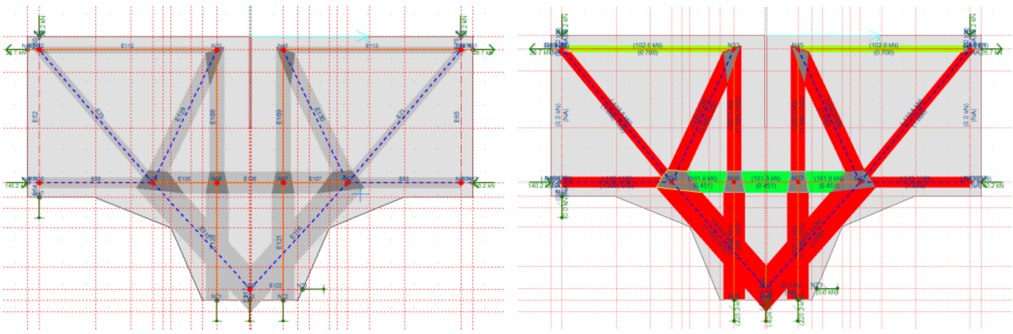

Figure 6: Model of partially split pier joint, modelled using Computer Aided Strut-and-Tie (CAST) software.

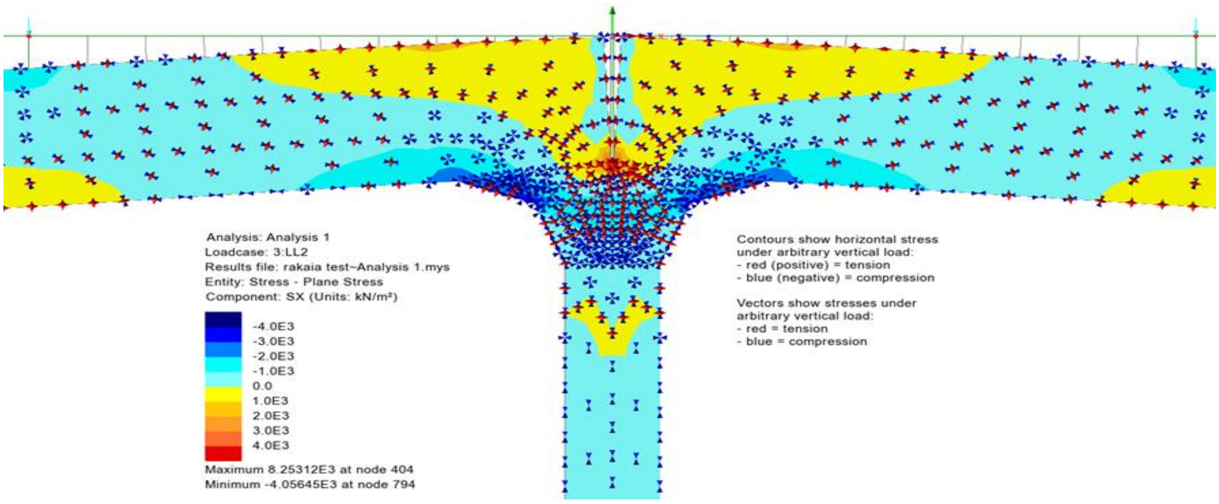

Figure 7: LUSAS model illustrating magnitude of stress vectors. Shading indicates the magnitude of the horizontal stress component.

Following test loading, SRM, and detailed inspections, the following hierarchy of failure of the bridges were able to be determined:

1. Increased cracking of the beam-column joints, and yielding of linkage bars and diaphragm reinforcement. Typical crack patterns are shown visually in Figure 8. The size of these cracks are expected to be minimal (typically <0.8mm) due to stiffness of the beams.
2. Cracking and spalling of the upper sections of pier columns.
3. Pull out of the vertical bars within the pier-columns, causing increased beam rotation and deflection.
4. Increased cracking in the soffit of the beams and yielding of the beams at midspan.
5. Continued deflection of the superstructure until the concrete kerbs and handrails rupture, transferring the entire compression force into the deck slab.
6. Considerable further deflection of the ductile reinforced concrete beams, prior to ultimate collapse. The beams are expected to gain around 25% additional capacity due to strain hardening of the reinforcement, with further additional strength due to the contribution of the concrete handrails and upper reinforcing steel in the deck

Partial continuity of reinforcement through the bridge joint provided a capacity improvement from 87% to around 100% HPMV capacity, allowing a significant increase in the gross mass of vehicles over these bridges. However, the beam/column joints were identified as highly stressed, with some reinforcement being at or near yield under service loads. In particular, the horizontal linkage bars were identified as being vulnerable to fatigue cracking. This could occur at any stage in the hierarchy above, although is more likely earlier on, while the beam column joints are relatively stiff. Failure of these bars is not necessarily critical to vehicle loading, but would significantly reduce the bridges seismic resilience.

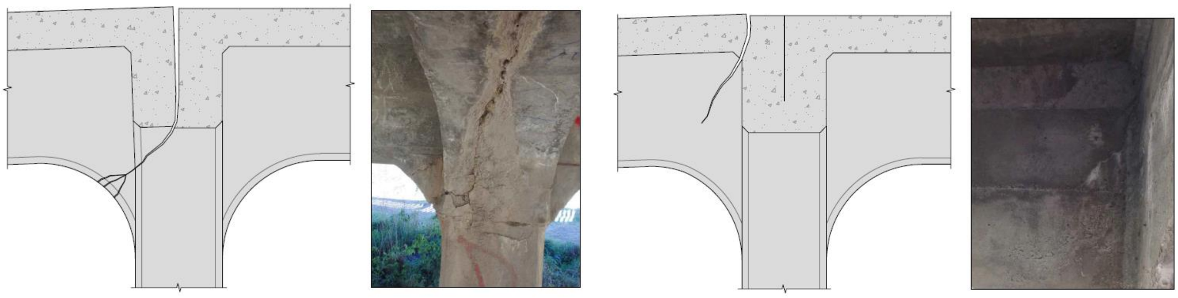

Figure 8: Critical joint failure mechanisms and examples of similar cracking on site

Focussed Response Monitoring

Ongoing monitoring systems

Based on the calibrated bridge analysis and identification of critical failure mechanisms, the following Structural Response Monitoring (SRM) systems were selected for ongoing monitoring of the bridges.

Weigh-in-Motion monitoring and enforcement

A Weigh in Motion (WiM) site was installed on the north approach of the Rakaia River Bridge in December 2014. The purpose of the WiM site is to provide real time information on the current loading profile of the bridge, to allow comparison with the increase once the route is opened to Full HPMVs. The WiM site also acts as a deterrent to vehicles overloading (by identifying vehicles for weight compliance checking), and provides considerable vehicle loading information to the NZ Transport Agency

Continuous accelerometers

Accelerometer outputs from the Rakaia River Bridge were monitored monthly by summarising the beam deflections into ‘bands’, and comparing these to vehicle weight ‘bands’ from the WiM site. This monitoring will specifically identify whether the proportion of larger beam deflections is increasing out of proportion with the heavy vehicle movements. The fundamental frequency of the beams is also being monitored to confirm this is not decreasing (as would be expected with damage to the joints or beams).

While the continuous accelerometer outputs have been valuable, the equipment and installation costs prohibit their installation on a larger proportion of the spans of these three bridges. More beneficial long term information may be obtained through alternative cost effective joint monitoring, as discussed below.

Continuous joint monitoring

Given the vulnerability of the beam-column joints on all three bridges; ongoing continuous monitoring systems are focussed on these critical zones to confirm whether any unrecoverable deformation is occurring. This monitoring will help protect against other damage mechanisms, which can only occur once significant damage to the joints has occurred. Joint monitoring can also be undertaken more readily and at lower cost than mid-span beam deflection using accelerometers, or other similar devices.

Permanent joint monitoring using displacement transducers has now been installed on both of the Rangitata River Bridges (as shown in Figure 9), with data transmitted via solar powered data loggers to the Opus network. If this monitoring is successful, this can be cost effectively extended to incorporate additional vulnerable spans. Joint damage will be monitored by separating peak joint displacements into ‘bands’. Site inspections will be triggered if the frequency of peak joint displacements increases out of proportion with the heavy vehicle movements. Trigger levels will also be set if the joint displacement is above critical thresholds, or if the frequency of joint displacement decreases, as this would be a sign that damage has occurred in the beam away from the joint. Overall thermal variations are being filtered out of the overall data when assessing vehicle displacements.

Figure 9: Permanent displacement transducer installed to monitor beam/column joint opening

Periodic test loading

Given the difficulty in tracing the response of a vehicle from the WiM to the accelerometer and displacement transducer outputs, and the complexity of effects from multiple vehicles; periodic annual test loading of the monitored spans is to be undertaken. This will ensure the accelerometers are calibrated, and will provide an independent confirmation that the overall fundamental frequency and stiffness are not altering, and deflection behaviour is remaining elastic. Moving forward, this requirement may be eliminated by comparing the WiM results from isolated heavy vehicles with the corresponding beam and joint displacements.

Periodic survey

Periodic six-monthly beam survey of all three bridges is being undertaken. This ensures the majority of the bridge spans have some form of coarse monitoring, given real-time monitoring will only occur on a very small proportion of the spans. The overall change in survey will be compared to that expected from thermal variation, and further investigations will occur if beam sagging increases beyond a specified threshold.

Concrete repairs

Concrete repair are currently being undertaken to critical areas of spalling on the Rakaia River Bridge. As part of the concrete repair contract, concrete is being removed within the diaphragms to identify the extent of corrosion of the lower bars. This will give an indication of the condition of the critical linkage bars across the piers, given the extent of leakage of water through the partially split joints. Installation of temporary strain gauges is also being considered, to confirm the actual stress in the linkage bars.

Develop Risk Mitigation System, Contingency Plans and Repair Methodologies

Given the existing risk and vulnerabilities identified on the Rakaia and Rangitata River Bridges, a risk assessment was undertaken following the general principles of the NZ Transport Agencies Risk Management standard (Z/44). This indicates that the overall residual risk to the bridges following the recommended mitigation measures was not significantly worse than in their previous state. However, significant risks were identified through the SRM process, particularly a vulnerability to corrosion and fatigue damage to the linkage bars over the non-split piers, and damage to the tops of the split piers. These risks are being partially mitigated through the current programme of concrete investigations and repairs. Linkage retrofit designs are likely to be prepared in the future to provide a retrofit solution that can be progressively installed along the bridge to further mitigate the risk of damage to these joints.

CONCLUSIONS

Whilst the SRM on the Rakaia and Rangitata River Bridges is still its initial stages, significant conclusions around the structural performance of the bridge have already been made. These include the following:

- The vehicle speeds required to cause resonance in the beams are around 130km/hr, and therefore increased impact loading through resonance is highly unlikely.
- The maximum span specific impact factor for these bridges is likely to be less than specified in the NZ Transport Agency Bridge Manual.
- Impact on these common T-Beam bridges increases with increased speed, as assumed by the NZTA Bridge Manual.

- The bridge spans are not acting as simply supported, and have considerable continuity across the partially split joints.
- Some continuity exists between superstructures even across the split piers, due to the interaction between the split pier columns.
- The inner beams displace significantly more than the outer beams. This is due to the additional stiffness from the kerbs and handrails, and the position that vehicles travel within their lane.
- Given the continuity through the spans and the reduced impact factor; the HPMV Evaluation capacity of the beams is > 100%.
- The beam/column joints are under significant stress under Class 1 and HPMV loading.
- This common form of beam/column joint is vulnerable to cracking and fatigue. This vulnerability is exacerbated for longer span bridges.
- The linkage bars in the beam column joints are under significant stress, and are at risk of fatigue cracking. This may result in a significant reduction in the seismic capacity and resilience of the bridge.
- The WiM data indicates that the level of overloading over the Rakaia River Bridge is relatively low. However, there is a risk that the level of overloading will increase once Full HPMVs are permitted over these bridges.

The five step SRM methodology has provide a cost effective framework for monitoring, which can be employed to other bridge structures in the future. This process is particularly beneficial where programmed strengthening or replacement is expensive, observed deterioration is less than would be expected from analysis, or where significant unknowns exist in the assessment and modelling of the structure.

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